CHAPTER 3. DESIGN CONSIDERATIONS

Section 3-1 Loads

As provided in the specifications, the falsework system must be designed to resist the sum of all dead and live vertical loads, plus an assumed horizontal load.

3-l.01 Dead Loads

Except as provided in the following paragraph, the dead load on a member is weight of the concrete, forms and reinforcing steel it is required to support, plus its own weight. The minimum value given in the specifications for the weight of concrete, forms and reinforcing steel is 160 pounds per cubic foot for normal concrete and 130 pounds per cubic foot for lightweight concrete.

When calculating actual beam deflection for comparison to the maximum deflection allowed by the specifications, the dead load on the member is the weight of the concrete only (see Section 3-2.01). For the dead load calculation, it is customary to use 150 pcf for normal concrete. For light weight concrete, use the actual value as determined from unit-weight tests.

Falsework must be designed to support the dead load weight of the entire superstructure cross section, excluding the weight of the railings, with the single exception of girder stems, and connected bottom slabs when deck concrete is placed more than five days after girder-stem concrete. In such cases the girder stem may be considered as self-supporting between falsework posts when the top slab is placed, provided the distance between falsework posts does not exceed four times the depth of the portion of the girder placed in the first pour.

The purpose of this exception is to reduce the design dead load on joists and stringers for box-girder structures in those cases where the girder stem (and the soffit slab as well where the soffit slab is loaded by the deck falsework) has gained sufficient strength to carry the weight of the top slab.

3-1.02 Live Loads

The design live load consists of a uniform load of 20 pounds per square foot applied over the total area supported; plus the actual weight of construction equipment, with the weight applied as a concentrated load at each point of contact; plus a uniform load of 75 pounds per linear foot applied at the outside edge of deck overhangs.

Judgment is required when investigating the effect of live loads caused by construction equipment, since instances will occur where equipment live load and concrete dead load are not applied at the same time. For example, when concrete placing equipment (such as a belt-spreader) is used in advance of the concrete front, there are two loading conditions. One loading condition will be uniform live load plus equipment live load plus an allowance for weight of forms and reinforcing steel, while the other will be uniform live load plus the normal dead load. Both conditions should be investigated.

For application of the uniform 20 psf live load; the total area supported includes the area of construction walkways that extend beyond the outside edge of the deck or the deck over-hang, However, the design load for all falsework supporting the walkway is the minimum vertical load, or 100 psf, as discussed in the following section.

The uniform load of 75 pounds per linear foot is applied only at the edge of deck overhangs. It is not applied along the edge of slab bridges or box girder bridges without overhangs, or at the edge of an interior deck construction joint.

<u>3-1.03 Minimum Vertica1 Load</u>

Regardless of the actual load imposed, the minimum vertical load (dead load plus live load) to be used in the design of any falsework member may not be less than 100 pounds per square foot, measured over the total area supported by that member. For application of this requirement, the term "total area supported" means, and includes, any area that is subjected to a dead load or a live load during any construction sequence.

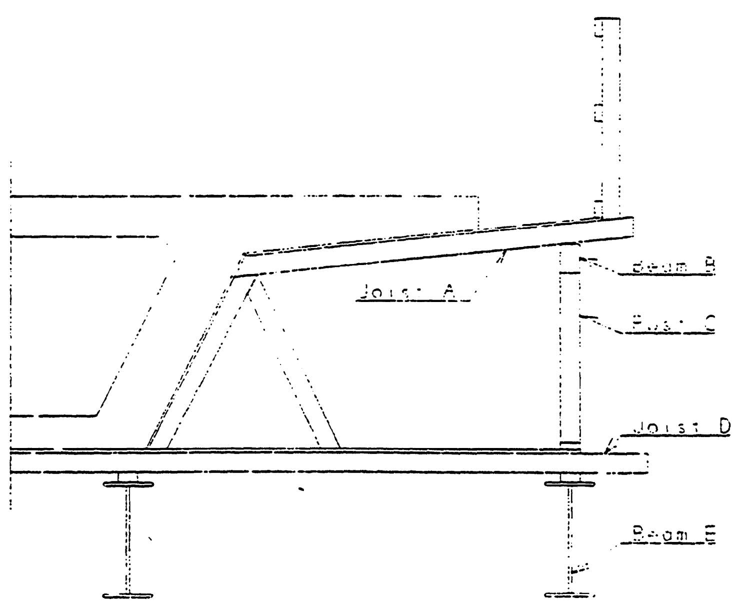
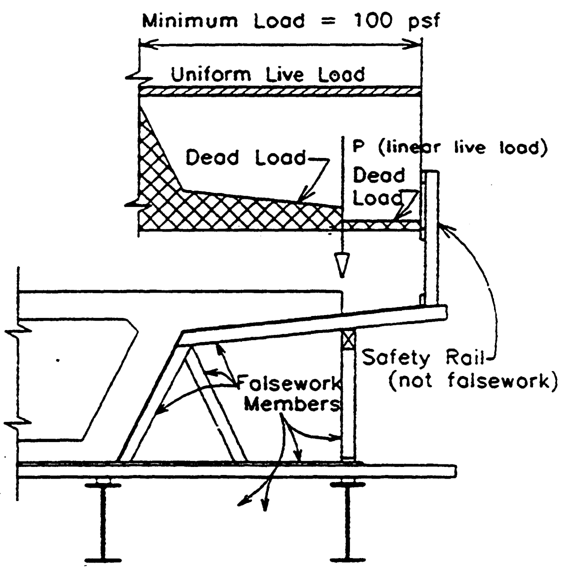


FIGURE 3-1

DESIGN CONSIDERATIONS

In the past, some confusion has occurred as to the design load for falsework supporting a construction walkway extending beyond the edge of deck or deck overhang. Refer to Figure 3-1, and note that Joist A, Beam B, Post C, Joist D and Beam E, and falsework members supporting Beam E, all see the construction walkway area as part of the "total area supported".

Figure 3-2 is a schematic portrayal of the various loads and load combinations specified for design of the deck overhang falsework. For the construction walkway itself (the walkwav planks or plywood) the design load is 20 psf. However, for the falsework members supporting the walkway, the design load is the minimum vertical load, or 100 pounds per square foot.



Dead Load - Actual weight of PCC, rebar, forms, falsework.

Live loads - Actual equipment load, uniform load of 20 psf.

P = 75 pounds/foot applied at edge of deck overhang.

Uniform live load is applied over the total area supported.

Minimum total design load for any falsework member is not less than 100 psf for live load plus dead

FIGURE 3-2

3-1.04 Loaded Zone for Deck Overhangs

Experience has shown that concentrated live loads, such as the load from finishing bridges and other miscellaneous equipment and materials not otherwise considered, can and do occur at or near the edge of a bridge deck during the concrete placing and finishing operations. In the case of deck overhangs, these loads may significantly increase the stresses in the overhang falsework support system. To account for the accumulated effect of such loads, the specifications include the requirement for a live load of 75 pounds per linear foot applied along the outside edge of all deck overhangs.

While the specified linear live load is a necessary design consideration for deck overhang falsework, its application to falsework components below the overhang support system will in the case of long falsework spans impose a design load that is unlikely to occur in actual. practice. To prevent an unrealistic loading condition for falsework members below the level of the bridge soffit, it is Division of Structures policy to limit the distance over which the specified 75-pound linear live load is applied to a loaded zone 20 feet in length measured along the edge of the overhang. The loaded zone will be viewed as a moving load positioned to produce maximum stresses in the falsework member under consideration.

The loaded zone concept for application of the 75-pound linear live load will be used when checking stresses in stringers, caps, posts and other members of the falsework system below the level of the bridge soffit in all cases where the falsework spans exceed 20 feet in length.

The loaded zone concept will also apply to the minimum vertical load (100 psf) on a construction walkway adjacent to the edge of the deck overhang. That is, when investigating stresses produced by the construction walkway load in falsework members below the level of the-bridge soffit in any case where the falsework spans exceed 20 feet, the 100 psf minimum load will be applied within, but not beyond, the 20-foot loaded zone as discussed above.

3-1.05 Vertical Design Load at Traffic Openings

The vertical design load for certain falsework members at traffic openings is increased to 150 percent of the load calculated in the usual manner. The application of this requirement is discussed in Chapter 8.

3-1.06 Horizontal Loads

The specifications require the "falsework bracing system" to be capable of resisting an assumed horizontal load applied in any direction.

As a design load, the assumed horizontal load is the sum of any actual loads due to equipment, construction sequence or other causes, plus an allowance for wind. In no case, however, may the horizontal design load be less than two percent of the total supported dead load at the location under consideration.

Keep in mind that the specified horizontal design load is an "assumed" load. Since it is an assumed load, it will not be equal, necessarily, to any actual horizontal load that may occur. Nevertheless, the falsework bracing system must be designed to resist the horizontal design load with the falsework in either the loaded or unloaded condition.

The requirement that falsework bracing must be designed to resist a horizontal load is included in the specifications to ensure both transverse and longitudinal stability. Falsework system stability is discussed in Chapter 5.

3-1.06 Wind Loads

The minimum horizontal design load (two percent of the total supported dead load) will govern the design for typica1 highway separation structures and otherstructures where the falsework height is less than about 30 feet. Depending on falsework configuration, wind may be a design consideration when height of falsework exceeds 30 feet, and wind loads will govern most designs where falsework height exceeds about 40 feet.

Determining the actual force exerted by wind on bridge false-work is a highly indeterminate problem due to the number of variable factors involved. These variable factors include the true wind velocity, the downwind width of the falsework system, the downwind distance between adjacent members, the drag or shape factor for the various members, the "solidity ratio" or percentage of solid-surface in a given gross frontal area, and the height of the falsework above the ground.

Although it would be theoretically possible, for a given false-work system, to establish values for each of these variables, such a procedure would be cumbersome and time-consuming at best, and (because of the subjective judgments involved) would not necessarily result in a more accurate answer than may be obtained by a simplified method. This, together with the need to ensure statewide uniformity in the wind load calculations, has led to the development of a specification which recognizes the effect of the more influential variables and assigns a coefficient to cover the others.

For the wind load calculation, the specification considers two general falsework types: (1) heavy duty steel shoring and steel pipe column falsework where the vertical members have a load carrying capacity exceeding 30 kips per tower leg or pipe column, and (2) all other falsework, including falsework supported by heavy duty shoring or pipe columns.

For heavy duty shoring and pipe column falsework systems; the wind load is the product of the wind impact area, a shape factor, and an appropriate wind pressure value. The wind impact area is defined as the total projected area of all elements in a tower face or falsework bent normal to the direction of the wind. The shape factor is included to account for the effect of wind drag forces on the members and, for heavy duty shoring, the effect of wind acting on members in the other three tower faces.

For all other falsework, including falsework supported by heavy duty shoring and pipe columns, the wind load is the product of the wind impact area and an appropriate wind pressure value. The wind impact area is the gross projected area of the false-work and any unrestrained portion of the permanent structure, excluding the area between falsework bents or towers where diagonal bracing is not used. In the specification context, the term "diagonal bracing" does not include flexible bracing systems. [1]

For all falsework types, the wind pressure value is a function of the height of the falsework. Wind pressure values, for each height zone, are tabulated in the specifications.

Except for falsework on driven pile bents, the height to be used in the wind impact area calculation is the vertical distance between the top of the component of the falsework system about which overturning rotation can occur and the bridge soffit. In the case of pile bents, judgment is required to determine the lower limit of the wind impact-area. If the piles are cut off and capped near the ground, the lower limit will be the plane at the pile cut-off elevation. If, however, the piles extend an appreciable distance above the ground, or above the water surface for structures over water, the entire height of the falsework (measured from ground or water surface to bridge soffit) should be used.

When calculating wind impact areas, keep in mind that formwork extending above the bridge soffit is not part of the wind impact area. [2]

Wind pressure height zones are always measured from the ground up regardless of falsework configuration.

Example problems illustrating the procedure to be followed when calculating the wind load on various falsework systems are included in the appendix.

3-1.06A Calculation of Wind Loads on Heavy Duty Steel Shoring

For wind acting on heavy duty steel shoring, the critical loading condition will occur when the wind force is applied at right angles to the tower faces. The effect of wind acting in

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[1] Flexible bracing includes cable, reinforcing steel bars, steel rods and bars, and similar members that do not resist compression.

[2] The specifications exclude formwork from the wind impact area under the assumption that when subjected to the design wind load, the forms would be blown off the falsework.

other directions need not be considered except in the case of temporary bracing installed during falsework erection and/or removal.

The horizontal design load produced by wind forces acting on heavy duty steel shoring is determined as follows:

1. From the tabulation in the Standard Specifications, select the wind pressure for each height zone.

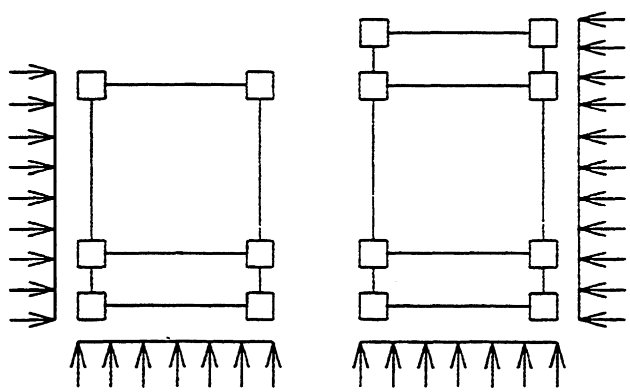
2. Multiply the selected wind pressure by the specified shape factor (2.2) to obtain the design wind pressure.

3. Calculate the total wind force per tower (WF) for each height zone by multiplying the design wind pressure by the total projected area of all the elements in the tower face normal to the applied wind.

   The following tabulation shows the projected area, in square feet per foot of tower height, for heavy duty steel shoring systems approved for use on State projects. In the tabulation; the projected area of the members has been adjusted to account for the effect of brackets, gussets and other minor components within the tower cross-section.

No Legs per face	WACO Shoring (ft^2/ft)	PAFCO Shoring (ft^2/ft)	WADCO Shoring (ft^2/ft)	HI-CAP Shoring (ft^2/ft)
2	2.00	1.75	1.93	1.64
3	2.50	2.25	2.43	---
4	3.00	2.75	2.93	---

No. of Legs = 3



No of Legs = 4

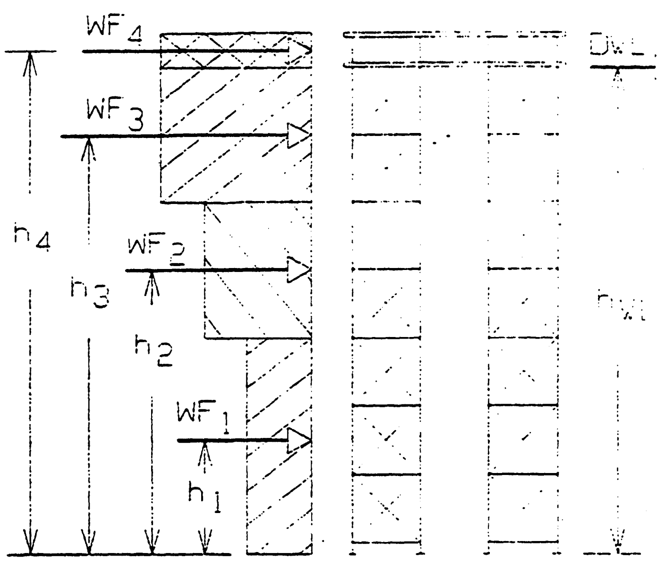
No. Of Legs = 2

4. For each height zone, calculate the overturning moment by multiplying the wind force (from step 3) by the distance from the base of the tower (top of the tower footing) to the center of pressure. Add the overturning moments for each height zone to obtain the total overturning moment.

5. Divide the total overturning moment determined in step 4 by the vertical distance between the tower base and a horizontal plane at the top of the highest falsework tower. The value thus obtained is the horizontal design load for wind (DWL) acting on the tower.

<u>3-l.06A(l) Analysis in the Transverse Direction</u>

Calculation of the horizontal designload for wind acting in the transverse direction is shown schematically in Figure 3-3.



Wind Load per Tower

1. **Shoring**

   $OTM_1 = WF_1h_1 + WF_2h_2 \div WF_3h_3$

2. **Supported falsework**

   $OTM_2 = 0.5\,WF_4h_4$

3. **Resultant wind load**

   $$DWL = \frac{OTM_1 + OTM_2}{h_{WL}}$$

WF = wind force acting on falsework in a given height zone.

DWL = design wind load.

FIGURE 3-3

Except as provided in the following paragraph, adjacent towers in the same falsework bent must each resist the design wind load because the upwind tower does not shield the downwind tower to any significant degree. This premise will be considered valid regardless of the distance between the towers, and will include those configurations where the space between abutting legs of adjacent towers is minimal. See Figure 3-4.

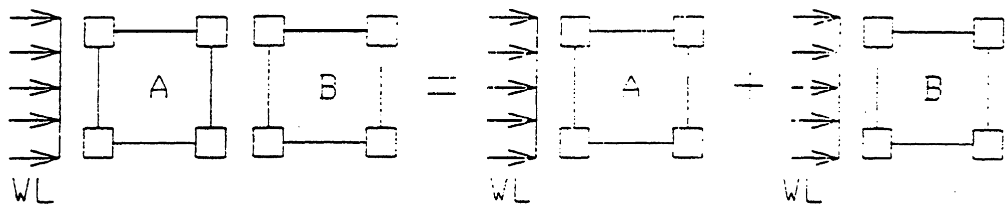


FIGURE 3-4

If the abutting legs of adjacent towers are connected, the total wind load for the two towers will be assumed as 1.5 times the design wind load acting on the upwind tower face. For analysis of the system, distribute one-half of the total wind load (or 75 percent of the design wind load) to each tower. See Figure 3-5.

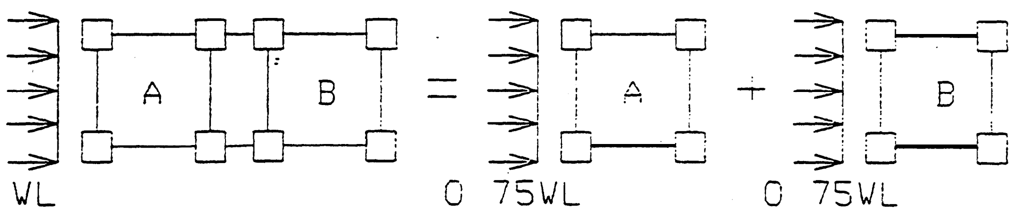
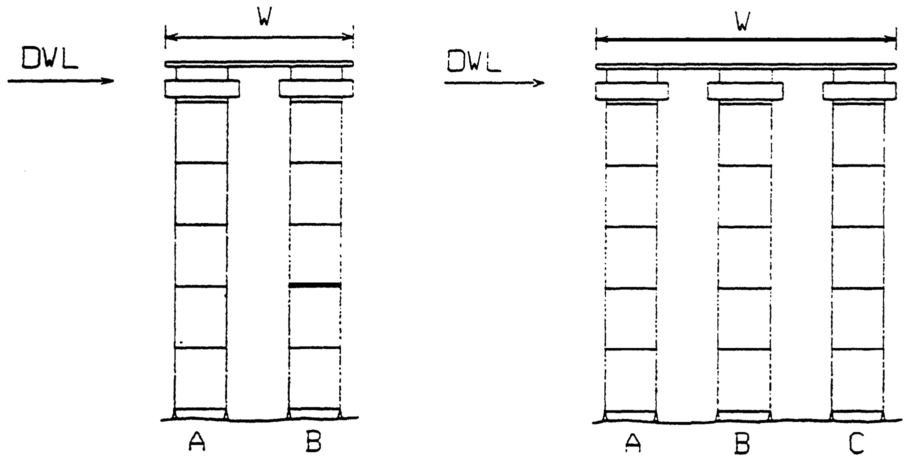


FIGURE 3 - 5

In addition to resisting the horizontal load produced by wind acting on the shoring towers, the falsework bracing system must resist the additional horizontal load produced by wind acting on elements of the falsework system (caps, stringers, joists, etc.) supported by the shoring. The design wind load on supported falsework is calculated by the wind impact area method. (See Section 3-1.06C.)

Refer to Figure 3-6 and note that for wind acting parallel to the falsework bent, the wind load on the supported falsework will be distributed to the individual towers in accordance with the following assumptions:

- For bents with two towers, one-half of the design wind load will be distributed to each tower.
- For bents with three towers or more, one-half of the design wind load will be distributed to the upwind tower and the remainder distributed equally to all other towers in the bent



0.5DWL 0.5DWL

0.5DWL 0.25DWL 0.25DWL

DWL = Design wind load on supported falsework system.

W = Downwind width of supported falsework system, determined in accordance with the procedure explained in Section 3-1.06C.

FIGURE 3-6

3-1.06A(2) Analysis in the Longitudinal Direction

For wind acting in the longitudinal direction or normal to the bent, the overturning moment calculation (wind load per tower) will be as depicted in Figure 3-3 for wind acting on the false-work towers. However, distribution of the load produced by wind acting on the supported falsework depends on the way the system is designed to resist longitudinal forces. Accordingly, when evaluating system adequacy, the load due to wind acting on the supported falsework should be distributed to the system in accordance with the discussion in Section 5-1.04, Longitudinal Stability.

3-1.06B Calculation of Wind Load on Pipe Column Falsework

For a pipe column falsework bent, the horizontal design load due to wind acting on the bent is the sum of the wind loads on the individual pipe columns in the bent. While this is obvious for wind acting normal to the bent, it is also the case for wind acting in the transverse direction (parallel to the bent centerline) because, typically, the columns are so widely spaced that shielding will not occur.

For adjacent columns where the downwind column is within the shielded zone, the applied wind force on the downwind column will decrease, but this will be offset by higher drag forces produced by increased wind turbulence. For this reason, it is Division of Structures policy to ignore any theoretical decrease in wind. load attributable to downwind shielding of adjacent pipe columns.

The design wind load is determined as follows:

1. Select the wind pressure for each height zone from the tabulation in the Standard Specifications.

2. For each height zone, multiply the selected wind pressure by the specified shape factor (1.0) to obtain the design wind pressure.

3. For each height zone, calculate the total projected area of the falsework bent. The total projected area is the sum of the projected areas (height of pipe column times diameter) of the individual pipe columns in the bent.

4. For each height zone, multiply the design wind pressure (from step 2) by the total projected area (from step 3) to obtain the wind force.

5. For each height zone, calculate the overturning moment by multiplying the wind force (from step 4) by the vertical distance between the point at the base of the pipe column frame about which overturning rotation will occur and the center of pressure.

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[3]See Section 3-1.07 for a discussion of shielding of downwind falsework members.

[4]The specified shape factor for pipe column falsework (1.0) has been adjusted upward from the true shape factor of 0.8 for circular sections to account for the effect of bracing and connections which are ignored in the calculations. This procedure is reasonable for typical pipe column bents where the bracing consists of small diameter steel rods or reinforcing steel, cable, or small structural shapes. However, in the event larger bracing elements are used, the projected area of the bracing components must be included in the total projected area of the falsework calculated in step 3. For this calculation, use a shape factor of 1.3.

Judgment is required when determining the point about which overturning rotation will occur. Typically, a pipe column bent is a rigid unit consisting of top and bottom cap beams, two or more columns, and internal diagonal bracing, all supported by a foundation system. Where adjustment is provide at the top of the bent, the lower cap or sill beam will be supported by corbels which distribute the load to the foundation. Where adjustment is provided at the bottom, wedges will be located between the sill beam (lower cap) and the corbels. In either typical case when overturning forces are applied, the bent will tend to rotate about a point at the bottom of the lower cap beam. For other configurations, the point of rotation should be determined as the lowest point in the system about which rotation can occur while the frame remains rigid.

6. Add the overturning moments for each height zone to obtain the total overturning moment.

7. Divide the total overturning moment by the vertical distance between the point of overturning rotation at the base of the frame (determined in step 5) and the top of the highest bent component. The value thus obtained is the horizontal design load for wind acting on the bent.

In addition to resisting the horizontal load produced by wind acting on the falsework members in the bent, the bracing must resist the additional load produced by wind acting on elements of the falsework system (stringers, joists, etc.) supported by the bent. The wind load on supported falsework is calculated by the wind-impact-area method discussed in the following section.

3-1.O6C Calculation of Wind Loads by Wind-Impact-Area Method

Except for heavy duty steel shoring and pipe column falsework bents, the design wind load to be applied to all types of bridge falsework, including falsework supported by heavy duty shoring and pipe column bents, is the product of an appropriate wind pressure value and the wind impact area of the falsework system under consideration. This method of determining the design wind load is commonly referred to as the "wind-impact-area" method.

The design wind load is calculated as follows:

1. Determine the value for W, which is the downwind width of the falsework system, or that portion of the system under consideration, measured in the wind direction.

For falsework supported by heavy duty shoring or pipe column bents, W will be the distance between the exterior beams or stringers. For all other falsework, W is the width of that portion of the falsework which supports a continuous cap or is connected by uninterrupted diagonal bracing.

2. Calculate the value for drag coefficient Q. From the specifications, $Q = 1.0 + 0.2W$, but not more than 10.

3. Calculate the wind pressure value for each height zone, using the wind velocity coefficient for that height zone as listed in the specifications and the value for Q calculated in step 2 above.

4. Calculate the wind impact area. The wind impact area as defined in the specifications is the gross projected area of the falsework and any unrestrained element of the permanent structure, excluding the area between falsework bents where diagonal bracing is not used. (Keep in mind that the term "diagonal bracing" as used in the wind impact area definition does not include flexible bracing.)

5. Calculate the total wind force for each height zone by multiplying the calculated wind pressure value by the wind impact area for that height zone.

6. Calculate the overturning moment for each height zone by multiplying the wind force (from step 5) by its distance above the point at the base of the falsework about which overturning rotation will occur. For this calculation, the wind force will be assumed as acting at the centroid of the wind impact area for the height zone under consideration.

   When determining the point about which overturning rotation will occur, keep in mind that an overturning failure occurs when a rigid element of the system, such as a braced frame or tower, rotates about the lowest downwind point of frame or tower support. Depending on the manner in which post or leg loads are distributed to the foundation, the point of overturning rotation may be at the top of a corbel or other load distributing member rather than at the bottom of the falsework system as a whole.

7. Add the overturning moments for each height zone to obtain the total overturning moment.

8. Divide the total overturning moment by the distance from the point at the base of the falsework about which overturning rotation will occur (determined in step 6) to the

top of the falsework post. The value thus obtained is the horizontal design load for wind acting on the falsework system.

For evaluation of system adequacy, the wind force will be applied parallel to and perpendicular to the longitudinal axis of the falsework bent. The effect of wind acting in other directions need not be considered in the analysis.

For wind forces (or a wind force component) applied parallel to the axis of a falsework bent, the calculated design wind load for each width (W) must be resisted by bracing within that width.

For wind forces applied perpendicular to the bent, resistance to the design wind load should be evaluated in the same manner as resistance to other longitudinal forces.

3-1.03 Effect of Shieldins on Wind Impact Area

When investigating the effect of wind acting perpendicular to a falsework bent, consideration may be given to the shielding provided by solid obstructions. Solid obstructions such as' abutment fills and pier walls will shield downwind falsework members to some extent, and thus reduce the wind impact area. The degree of shielding actually provided is speculative, however, and not easily determined. To ensure uniformity, the assumptions discussed in the following paragraphs will be considered as Division of Structures policy.

As wind blows around the end of a solid obstruction, the area over which the wind pressure is effective will increase inward on a 2:l ratio (downwind distance to inward distance) as shown in Figure 3-7. Falsework bents within the shielded zone will be considered as totally sheltered from wind forces-

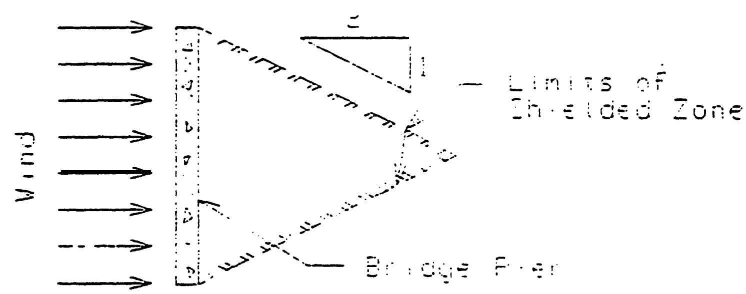

Figure 3-7

In the case of falsework bents which are partly shielded, the term "gross projected area of the falsework" will be interpreted as meaning the area of the bent that is outside the shielded zone. (See Bent A in Figure 3-8.) When checking such bents for stability, the total wind load may be distributed uniformly along the entire length of the bent.

In the case of adjacent bents which are fastened together to form a single, longer bent, the wind load may be distributed into the adjacent bent provided the bents are rigidly connected. Such bents will be considered "rigidly connected" if in the engineer's judgment, the connection is capable of transferring the wind forces. (See Bent C in Figure 3-8.)

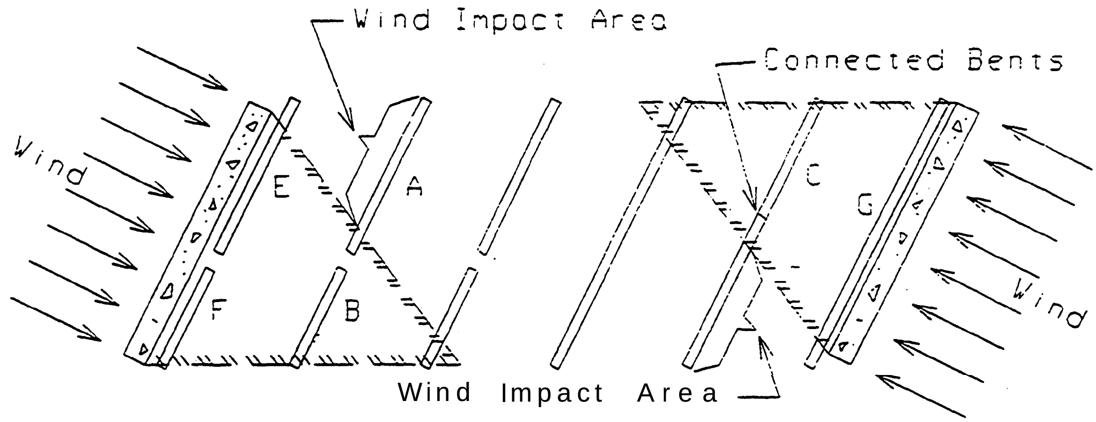

FIGURE 3-8

In the case of bents which are located immediately adjacent to a solid obstruction, the effect of wind may be neglected since the exposed area is relatively small. (Bents E, F and G.)

When investigating the effect of shielding, keep in mind that wind may blow from any direction. Falsework bents that are totally shielded from wind in one direction may be fully exposed when the wind forces are applied from the opposite direction.

Section 3-2 Deflection and Camber

3-2.01 Beam Deflection

When reviewing falsework drawings, a distinction must be made between the maximum deflection allowed by the specifications and the actual deflection under a given loading condition.

The maximum allowable deflection is calculated using the weight of all concrete in the superstructure cross-section (as though the entire superstructure were placed in a single concrete pour) and is limited to l/240 of the span of the falsework beam. This limiting value is included in the specifications to ensure a certain degree of rigidity in the falsework and thereby minimize distortion of the forms as concrete is placed.

Actual deflection is the deflection that occurs as the falsework beam is loaded. Calculating actual deflection is the engineer's responsibility, since it is used in determining the amount of falsework camber required. [5]

When calculating actual deflection, it is necessary to include the weight of forms and falsework supported by the beam, the weight of the beam itself, and the weight of the concrete. In addition, consideration must be given to such factors as the sequence of construction and the depth of the superstructure when two or more concrete pours are involved.

The specifications do not include a limiting value for live load deflection, since live loads are of a transient nature. However, when a bridge deck finishing machine is supported at the outer edge of a cantilevered deck overhang/particular care must be taken to prevent excessive deflection of the deck overhang support system. Unless special precautions are taken, the concentrated load due to the weight of the finishing machine may cause the deck overhang to deflect appreciably with respect to the remainder of the deck surface, and this will decrease bridge deck thickness. and reduce reinforcing steel cover, both of which are detrimental to the completed structure.

The applicable specification in this case is the general requirement that falsework must be designed and constructed so as to produce in the finished structure the lines and grades shown on the plans. To ensure compliance with this general requirement, it is Division of Structures policy to add the deflection due to the weight of a deck finishing machine to the deflection due to the weight of the concrete, and the sum of these two deflections should not be so large as to adversely affect the character of the finished work. Obviously, this will require engineering judgment. The important point is that the weight of the finishing machine be considered, and the total deflection limited to a realistic value.

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[5] The specifications require the contractor's design calculations to show the "stresses and deflections in load supporting members." Said deflections, which may not exceed l/240 of the span, are calculated using the theoretical weight of the concrete supported by the member: There is no requirement that the contractor furnish "actual" deflections.

3-2.01A Negative Deflection

Depending on the concrete placing sequence, negative (upward) deflection may occur where falsework beams are continuous over a long span and a relatively short adjacent span. This condition (negative deflection at the end support) is an indication of system instability and must be considered in the falsework design. If beam uplift cannot be prevented by loading the short span first, the end of the beam must be restrained or the span lengths changed. Division policy does not permit approval of any design where theoretical beam uplift will occur under any loading condition.

When falsework beams are considerably longer than the actual falsework span, the beam cantilever extending beyond the point of support will deflect upward as the main span is loaded. The falsework design must include provisions to accommodate this upward deflection. The usual method uses a filler strip (often called a "sleeper") on the main span only, which allows free movement of the beam cantilever.

3-2.02 Camber

The term "camber" is used to describe an adjustment to the profile of a load-supporting beam or stringer so the completed structure will have the lines and-grades shown on the plans. In theory, the camber adjustment consists of the sum of the following factors:

- Anticipated total deflection of the falsework beam under its own weight and the actual load imposed.
- Difference between beam profile and profile grade, also called vertical curve compensation,
- Difference between beam profile and ultimate super-structure deflection curve,
- Difference between beam profile and any permanent or residual camber to remain in the structure for its useful service life.

In structures with parabolic soffits, an additional adjustment may be required to account for the difference between beam profile and soffit curvature.

When falsework beams are relatively short, the theoretical adjustment due to vertical curve compensation, ultimate super-structure deflection and desired residual camber will be small and may be neglected. As falsework spans increase, these factors become increasingly significant and must be considered along with beam deflection.

More than any other single factor, the satisfactory appearance of a completed structure will depend on the accuracy of the camber used in the falsework construction. Obviously, good judgment will be required, particularly in determining the amount of camber to be used to compensate for anticipated dead load falsework deflection.

In general, the weight of the top slab of conventionally reinforced box girders should be omitted when calculating camber, since additional stringer deflection as the top slab is placed usually is insignificant. In the case of cast-in-place prestressed construction, falsework span length may be an important consideration. In such structures, judgment will be required as to the relative stiffness of the girder stems, and whether and how much they will resist additional deflection as the top slab is placed.

3-2.02A Camber Strips

When directed by the engineer, the contractor is required to furnish camber strips to compensate for beam deflection and to adjust the final profile for vertical curve, superstructure deflection and residual camber considerations.[6]

When to require camber strips is a matter of engineering judgment. As a general rule, camber strips-are not necessary unless the total camber adjustment exceeds about l/4-inch for beams supporting the edge of the girder soffit or deck overhang and/or about l/2-inch for beams at interior locations.

Because camber strips are an incidental part of the falsework system, their installation seldom receives more than cursory attention. Casual treatment of camber strip installation can result in an unforeseen, and undesirable, loading of the falsework beam. For example, a camber strip placed at a distance away from the centerline of a steel beam may induce torsional stresses that were not considered in the falsework design. Undesirable torsional stresses may be induced in beams supporting falsework for structures having steep cross slopes, even if the camber strip is properly placed along the beam centerline. For such structures, unless the vertical dimension of the camber strip includes an allowance to compensate for cross fall, the joist may bear on the edge of the beam flange rather than on the camber strip itself.

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[6] Camber strips are lengths of wood cut to the shape of the camber curve and fastened to the top of the falsework beam or stringer. Typically, camber strips will be secured by nailing to the top of timber members, or by banding in the case of, steel members.

To ensure proper design and installation, camber strips must conform to the following criteria:

- The width of the camber strip shall be not less than l-1/2 inches.
- Structure cross slope, allowable wood crushing, and joist deflection shall be considered when determining the height of the camber strip. The minimum height of the camber strip shall be such that the joists will not come into contact with any part of the falsework beam under any loading condition.
- Camber strips shall be centered along the longitudinal centerline of the falsework beam.
- Camber strips shall not extend onto the unloaded portion of a trailing beam cantilever.

The allowable stress for perpendicular-to-the-grain loading has been increased from 450 psi to 900 psi for camber strip bearing loads. When the applied load produces the maximum allowable stress, camber strip deformation due to crushing should not exceed about l/8-inch.

If the amount of camber is large, as: would be the case, for example, where a curved bridge soffit is supported by a long falsework beam, the camber strips should be braced or built up with wide material to avoid lateral instability. The use of laterally-unsupported tall, narrow camber strips should not be permitted.

3-2.03 Horizontal Deflection

Although the specifications do not include a limiting value for horizontal deflection, such deflection will be negligible in any falsework system where horizontal forces are resisted by bracing. Accordingly, horizontal deflection need not be considered in any case where the horizontal design load is resisted by a properly designed bracing system. This includes external bracing systems where the use of external bracing is necessary to prevent overturning.

Horizontal deflection will be a consideration when the horizontal design load is resisted by bending in a falsework member. This situation occurs when falsework is supported by driven piles that extend above-the ground surface. When evaluating the adequacy of pile bents, it is necessary to combine bending and vertical load stresses to obtain the actual stress.

The procedure for evaluating the adequacy of falsework pile bents is discussed in Chapter 7.

Section 3-3 Miscellaneous Considerations

Section 3-3.01 Beam Continuity

Because of the sequential, and sometimes unpredictable, manner in which falsework loads are applied, beam continuity is an uncertain design condition. To accommodate this uncertainty, it is Division of Structures policy to assume the continuous beam condition when continuity will act to increase loads or stresses, but not otherwise.

For example, the simple span condition will be assumed when calculating positive bending moments in joists, stringers, and similar continuous members; however, full continuity will be assumed when calculating negative bending moments in these same members, Assume full continuity when calculating the beam reaction on interior supports under continuous falsework members, but assume the simple span condition when calculating the reaction at the end support.

In a framed bent, continuity must be considered in any case where stringer loads are applied within the cap span rather than directly over the supporting post to ensure that allowable post loads are not exceeded.

Continuous caps are often supported by two or more towers in a heavy duty shoring system. If leg loads are unequal, the resulting differential leg shortening will cause a redistribution of beam reactions and a corresponding change in the magnitude and location of maximum cap bending stress.

When beams are continuous over two or more spans, beam uplift can occur in adjacent unloaded spans when concrete is placed in one span (see discussion in Section 3-2.01A, Negative Deflection).

The engineer will be expected to recognize these and other cases where the effect of beam continuity must be investigated to prevent the overstressing of any falsework member, or instability in the falsework system,

3-3.02 Construction Sequence

Unless a concrete placing sequence is shown on the plans, the falsework drawings must include a placing diagram showing the proposed placing sequence and the location of all construction joints. If a placing schedule or sequence is shown on the plans, no deviation is permitted and the falsework must be designed and constructed to accommodate the planned placing sequence.

The location of transverse construction joints in the bridge superstructure is an important falsework design consideration. If a construction joint is located near the mid-point of a falsework beam, the initial concrete pour on one side of the joint will deflect the beam as the concrete dead load is applied. Later, as concrete is placed on the opposite side of the joint, additional beam deflection will occur. The additional beam deflection leaves the first concrete placed unsupported, and this can result in unanticipated detrimental stresses and even cracking in the permanent structure. To avoid this condition, transverse construction joints in the bridge superstructure should be designed and constructed in such a manner that subsequent pours will not produce additional stresses in the concrete already in place.

In many cases the exact location of a construction joint is not critical, and the joint can be moved a few feet in either direction to accommodate the falsework design. The important point, however, is that joint location with respect to falsework beam span be considered in the falsework design, thus avoiding a problem during construction.

When relatively long falsework spans are used to support T-beam structures, the added weight of the deck concrete, which often exceeds the weight of the stem, loads the stem as well as the falsework as the deck concrete is placed. This can produce stresses of considerable magnitude in the concrete and reinforcing steel in the girder stem.

To prevent overstressing of concrete and reinforcing steel in the girder stems, the specifications limit the length of falsework spans used to support T-beam structures to 14 feet plus 8.5 times the depth of the T-beam girder. In the specification context, the term "depth of the T-beam girder" means the distance between the top of the deck and the girder soffit.

3-3.03 Friction

Friction as a means of resisting-opposing horizontal forces is a very intangible factor; accordingly, the use of friction for this purpose should be considered with caution. When friction is used, the coefficient of friction should be assumed as 0.30 for all contact surfaces.

In general, friction may be considered as resisting the tendency of one member to slide over or across another member, provided frictional resistance is actually developed under the loading condition being investigated.

Do not consider frictional resistance in any case where the dead load is not applied uniformly through all stages of construction, or where continuity would reduce the load acting on a support under a nonuniform loading condition.

Do not consider frictional resistance as contributing to the lateral stability of beams or stringers. If flange support is required, the method of support must be positive and independent of any theoretical frictional resistance.

Except as otherwise provided in the following paragraphs, do not consider friction as contributing to the resisting capacity of any connecting device unless the device is specifically designed and marketed as a friction-type connector.

Heavy-duty commercial and/or noncommercial C-clamps having a torque-tightening capacity of 90 foot-pounds or more may be used as connecting devices in accordance with the criteria included in Falsework Memo No. 5 in Appendix C. Note, however, that C-clamps are not to be used at any location where they are exposed to vandalism, such as at the bottom of posts or along sill beams.

Approved clamps may be used to resist horizontal forces up to 3000 pounds per clamp. For falsework adjacent to traffic, approved C-clamps by be used to resist the 500 pound force applied in any direction at the top of the falsework post as well, except that a clamp used for this purpose may not be used to resist any other force.

Approval of C-clamp installations shall conform to the following procedure:

The location of the clamps must be shown on the falsework drawings. All use restrictions in Falsework Memo No. 5 must be met.

- The falsework drawings must include a note requiring all clamps to be torqued to 90 foot-pounds minimum.
- For commercial clamps, the contractor must furnish a catalog cut or manufacturer's technical data sheet describing the clamp in sufficient detail to verify compliance with product criteria in Falsework Memo No. 4.
- For noncommercial clamps the falsework drawings must include a sketch showing the dimensions of the clamp. The clamp must comply with Falsework Memo No. 5.

3-3.04 Prestressing Forces

When cast-in-place prestressed structures are stressed, the initial stressing produces an upward deflection in the positive moment area, and the resulting redistribution of vertical forces transfers the superstructure dead load from the false-work to the adjacent abutments and columns.

Dead load redistribution due to longitudinal prestressing will not be a falsework design consideration unless stage construction is required, as will be the case, for example, with continuous structures having hinged connections. For such structures, prestressing will reduce the dead load on the falsework near the center of the suspended span and increase the load on the falsework at the hinge. The forces involved in the dead load redistribution are of considerable magnitude, since up to 3/8 of the total suspended span dead load may be transferred to the falsework at the hinge. The load due to dead load transfer must be added to the dead load calculated in the usual manner to obtain the total dead load for the falsework design at the hinge support.

If the dead load hinge reaction (the load applied to the canti-lever span by the supported span) is not shown on the contract plans, it may be obtained from the designer.

The effect of transverse prestressing is also a consideration. If the structure is designed to include transverse prestressing of decks or caps, the special provisions will include the stressing sequence, and the falsework must be designed to accommodate this sequence.

3-3.05 Long-term Superstructure Deflection

As discussed in the preceding section, falsework at a hinge must be designed to carry the additional load imposed when the super-structure is stressed. Depending on such factors as the length of time the falsework is to remain in place and the method and sequence of falsework removal, long term deflection of the bridge superstructure occurring subsequent to stressing may be a design consideration as well.

Long term superstructure deflection will begin as soon as the structure is stressed. As deflection occurs, a portion of the dead load initially transferred to the falsework at the hinge will be carried back to the falsework near the center of the span. The amount of dead load carry-back is a function of time and is not easy to predict, but this should not present a problem in most cases because the load carried back cannot exceed the load originally resisted by the falsework.

However, if falsework is removed in stages, field engineers should be aware that part of the redistributed load will be carried back with time, and that components of the falsework system remaining in place near the center of the span will be subjected to a gradually increasing load as superstructure deflection takes place. Accordingly, dead load carry-back may be an important consideration when evaluating the adequacy of a given falsework removal sequence. (See also Section 9-l.14A, Stage Construction.)

3-3.067 Falsework at Deck Overhangs

For box girder structures with cantilevered deck overhangs, the normal two-stage construction sequence results in differential loading of the exterior and first interior falsework beams. The differential loading condition is exacerbated if the exterior girder is. also sloping outward at the top, as is usually the case. Depending on beam size and location, differential loading may result in differential beam deflection, causing the exterior girder stem to rotate. Girder rotation may occur during the girder stem pour or during the deck pour, or during both pours.

Refer to Figure 3-9, and note that during the girder stem pour, Beam B may deflect more than Beam A, in which case Point D will move upward relative to Beam B. This upward movement at Point D causes the girder-stem form to rotate inward. Inward rotation will affect alignment and grade at the top of the girder stem, but in most cases this is not serious since any required adjustment can be made before the deck pour. However, the effect of differential beam deflection during-the girder stem pour should be investigated as a precautionary measure to determine whether any adverse consequences will occur.

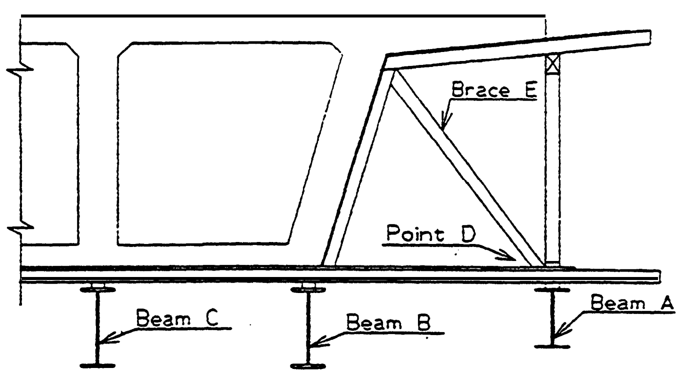

FIGURE 3-9

A more serious situation may develop during the deck pour where the weight of the deck overhang may cause Beam A to deflect more than Beam B, particularly if Beam A is a smaller member as is sometimes the case. This differential deflection causes a downward movement at Point D relative to Beam B, which pulls Brace E away from the girder stem form panel and leaves the sloping exterior girder unsupported. The weight of the unsupported exterior girder produces an outward rotational moment which, if not resisted, will cause unanticipated torsional stresses in the concrete and reinforcing steel at the girder base. This condition (outward rotational moment) is exacerbated by the weight of the deck concrete on the inside of the exterior girder.

The load applied to the exterior and adjacent interior false-work beams during the deck pour should be investigated in all cases where the depth of a box girder structure having sloping exterior girders exceeds five feet. When the applied loads result in differential beam deflection of sufficient magnitude to cause the exterior girder support system to become dysfunc-tional, the falsework design must include a means to resist girder rotation. Division of Structures policy requires the method by which this is accomplished, such as tie-backs to the base of the adjacent interior girder, to be shown on the falsework drawings.

For various reasons, contractors occasionally follow a three-step construction sequence in which the soffit slab is placed independently of the girder stems. When the soffit slab is placed as a separate concrete pour, Beam C will deflect more than Beam B and there will be no appreciable deflection at Beam A. These differential beam deflections will result in an upward movement at Point D relative to Beam B. Additionally, joist continuity will tend to lift the soffit joist at Point D as concrete is placed. Upward movement at Point D resulting from these factors will rotate the girder form inward. If these movements are appreciable, it may be necessary to realign the form before placing stem concrete.

3-3.07 Concrete Decks on Steel Girders

Section 55-1.05 of the Standard Specifications includes special requirements for falsework supporting the concrete deck on steel girder bridges. These requirements are included to control the manner in which falsework loads are applied to the steel girder, and thus prevent undesirable distortion of the permanent structure.

Horizontal loads applied to the girder flanges by the falsework will produce a torsional moment in the girder. To prevent possible overstressing of the permanent diaphragm connections, the falsework design must include temporary struts and/or ties to resist the full torsional moment and to prevent appreciable relative vertical movement between the edge of deck form and the adjacent steel girder.

Additionally, the falsework must be so designed and constructed that any loads applied to the girder web will be applied within six inches of a flange or stiffener. The applied loads must be distributed so as to prevent local distortion of the web.